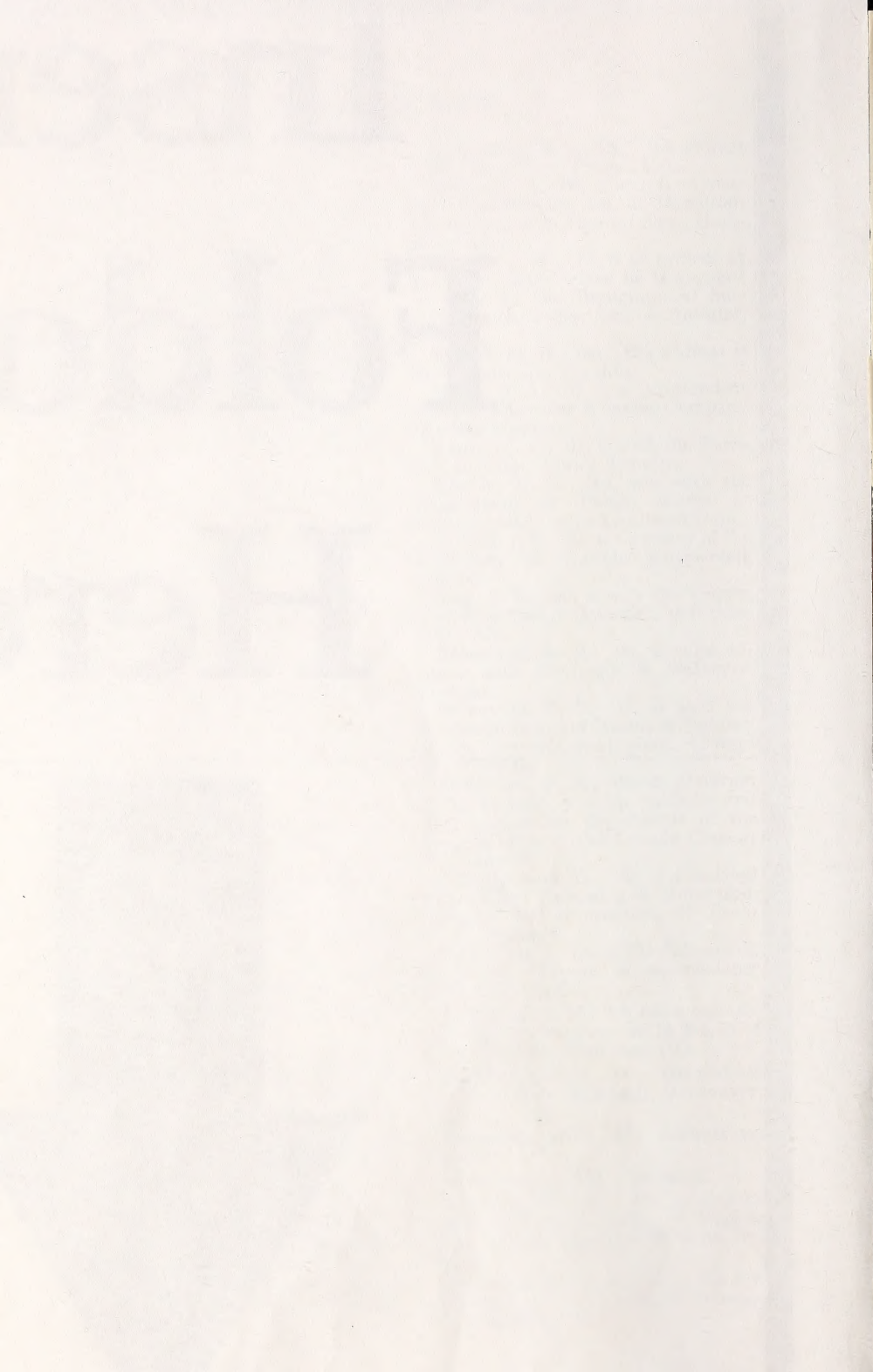


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WIND BRACING IN TALL BUILDINGS

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The question of wind bracing in tall steel buildings of the skeleton type has evoked considerable discussion as to methods and design in the last few years. One class of structural designers has made ample provision in the steel frame for lateral stiffness based upon direct computations of wind stresses; while on the other hand, another class has ridiculed all mathematical treatment, and used "rule of thumb" methods for design. The exact treatment of this part of building design would lead to hopeless intricacy whether diagonals, knees, or portals are employed to transmit the lateral wind pressures. Although an exact determination of stresses is difficult, if not impossible, it does not follow that a mathematical analysis is useless and without any rational basis. The investigation of the stability of a steel frame building subjected to wind pressures must be solved in accordance with the fundamental laws of mechanics and strength of materials as determined from the theory of flexure.

In very tall buildings the calculation of wind stresses and the design of frame work to afford the most rigid and economical resistance to them, while conforming to architectural requirements, are matters which sometimes have relatively large dimensions in both directions, the inertia of the building, the stiffness of walls and floors and the inherent strength of the beam and column connections generally used, may afford sufficient rigidity, and precise calculations are often omitted.

When the building is comparatively tall and narrow the elasticity of the steel frame makes it necessary to make definite provision against horizontal forces in at least one direction, the steel skeleton being usually considered as a vertical cantilever exposed to horizontal wind forces in every direction, in the upper part at least, and sometimes for the entire height of one or more sides not protected by adjacent buildings.

In the usual case of a transverse bent of several columns it is impossible with the means of analysis at present available to deter-

mine how much of the vertical loading on these columns is transferred by the wind pressure from each column at windward side to each column on leeward side, although it is absolutely certain that such a redistribution takes place.

This condition makes it impossible to determine with certainty both the horizontal shears transmitted through the columns, and the bending moments to which columns and girders are subjected.

The design of the steel frame of high buildings must certainly include as one of its main features suitable effective provisions for wind loads whose effects run through the entire height of the structure and by which the transferred loads influence materially the foundation pressures.

WIND PRESSURES

Wind pressures on small surfaces have been recorded as high as 50 pounds per square foot, but it is doubtful if such pressures exist on large surfaces, as sides of buildings. A few of the influences which affect the intensity of wind pressures will be summarized briefly.

Effect of Altitude on Wind Pressures.—Since air is lighter in greater altitudes the wind pressure will also be less. In addition to and quite distinct from this, it is also clearly recognized that the pressure varies as the distance above the general level of the surrounding territory, increasing towards the top of the structure.

Effect of Temperature Changes.—Mr. W. H. Whitten made a careful study of the effect of temperature changes, and his conclusions were:

(1) That the pressure increases as the temperature drops, if the velocity is constant and barometer readings normal.

(2) That the pressure increases and diminishes as the barometer rises and falls.

Effect of suction.—Rapidly moving air produces a suction on the leeward side, there being a partial vacuum developed in the constant vortex or eddy on this side. Professor Albert Smith reduces the following laws with regard to suction.

(1) Increase in height tends to increase relatively the average amount of suction on the leeward wall.

(2) The relative amount of air flowing around the end of building increases as the height increases.

Effect of Gusts.—The maximum wind pressures are of short duration, probably only for a second, but sometimes occur at intervals, and thereby tend to set up an oscillation in structures they are capable of influencing. To some extent the wind impulses are absorbed in overcoming the inertia of the structure, being exhausted in internal work. Should the wind come in gusts, overcoming the inertia of the building, and causing the building to oscillate, and if the oscillations and wind impulses act together, the vibration would be increased and there would be danger of the building collapsing. Existing experiments tend to indicate the period of vibration of a tall building is several seconds (about 2 to 4 seconds) and thus the likelihood of such an agreement taking place does not seem possible.

It seems advisable that the increased pressure due to gusts should be considered in designing tall building frames.

EFFECT OF WIND ON BUILDINGS

The effect of wind pressure tends to overturn the building on its base. In tall narrow buildings this overturning effect must be investigated, and if necessary, provision made for it by anchoring the columns of the foundations. Common specifications state that if the overturning moment exceeds 75 per cent. of the moment of stability, anchorage for the columns must be provided. From the footing plan the column loads and their point of application can be taken off and by the principle of moments the point of application of the resultant weight can be found. Knowing the magnitude and point of application of the resultant weight, the moment of stability can be readily computed.

It may give way in a horizontal direction under the influence of shearing forces either by lateral displacement from its foundations or by buckling of its various members on account of their weakness to resist these forces. If the buildings were safe against overturning it would ordinarily be safe against sliding bodily, and for the sliding tendency to be considered the width of the base must be two-fifths or more of the height. The tendency to shear the connections is one of the most important features of wind pressure. Special attention should be given the column splices and the connections of floor girders to columns. The details should be sufficient to develop the strength of the main part of the member and to give the required rigidity at the joints.

Deflection in a bent to leeward due to wind pressure will be the result of a combination of three actions—first, the elongation of the tension columns and the shortening of the compression columns; second, the deflection of the floor girders; third, the deflection in the columns. Factors affecting the amount of deflection are the sizes of the columns, their unsupported heights, and the number of columns in a cross section.

In buildings in which the ordinary beam connections are used with no special bracing, as the connections are capable of transferring only small bending moments, the deflections will be large. It will be seen that if the girders cannot take care of any bending at the connections, and there is no other factor to resist distortion, then the frame must simply close upon itself under lateral forces.

The few experiments on the deflection of tall buildings seem to reveal the fact that the actual deflection is less than the computed. Experiments on a seventeen story building by Dr. Melick show that walls and partitions reduce the deflection about 40 per cent., and they seem to be secondary and not primary agencies in resisting distortion to wind. His conclusions were that, when proper account is taken of the velocity and pressure of the wind, vibrations and deflections computed on the basis of the steel frame resisting all the pressure, will be only slightly in excess of those actually existing.

A twisting as well as a direct deflection will be induced in the building if the bracing is not symmetrically disposed with respect to the centre line of the building.

NECESSITY OF WIND BRACING

The New York building by-law requires that all buildings exposed to the wind (except those under 100 feet. in height in which the height does not exceed four times the average width of the base) be designed to resist wind pressure. This may be regarded as representative of the type of building requiring no bracing.

In structures of the skeleton type which have relatively large dimensions, the dead weight of the building, the stiffness of the walls, partitions and floors, and inherent strength of column and girder connections may afford sufficient rigidity, and precise calculations are often omitted. The relative merits of each of the elements mentioned in the above statement for resisting wind pressures will be discussed in detail.

The stability of a steel frame building must depend upon its dead weight or its steel framework. The weight of the building affords some resistance but it is not altogether a dependable quantity. The greater the weight the greater the moment of stability. The moment of stability may be computed as previously outlined under "Effect of Wind on Buildings."

Distortion may be resisted by the compressive strength of the curtain walls, but as they are usually cut up by numerous windows they cannot be relied upon to act in conjunction with the steel frame.

Partitions as ordinarily constructed are too thin to resist shearing stresses, and their location is never definitely and finally known to the designer, in as much as they are liable to removal at the will of the tenant. They are often omitted permanently on the first floor and in many cases are not put in the upper floors until rented, when the tenants' wishes as to subdivision can be learned. Mr. Purdy states that if partitions be omitted on one floor it is nearly as bad as if they were omitted on several. It is doubtful if much reliance can be placed upon partitions for cross framing to transfer shearing stresses into vertical reactions.

Floors are effective in producing a redistribution on wind stresses from the windward to the interior and leeward columns. Tile arches assist the wind bracing of the structures because they fill the total depth of the steel beams, and act as horizontal bracing for the entire structure. In comparison with this, concrete floor slabs rest only upon the upper third of the beams and are usually one-third the depth of tile arches. They cannot efficiently transmit the horizontal stresses, and, by reducing the efficiency of the floors as braces, increase the amount of steel necessary to provide against horizontal stresses.

There is also some resistance to lateral strains combined through the various connections of the beams, girders and columns, but it is proportional to the details employed in such connections.

The above considerations will not readily admit of calculations

and in using them much will depend upon the experience and judgment of the designer.

It may be said that partitions, walls, floors, etc., as ordinarily constructed, cannot be relied upon to act in conjunction with the steel frame in resisting wind forces, and there will be a limit beyond which the steel frame can take up such forces. Modern steel office buildings as built to such great heights, especially in proportion to their width, are so destitute of the ordinary means of resisting wind forces that it is necessary to give the subject much more consideration. The designer cannot rely upon the elements of strength, uncertain in value and irreducible to calculation. He must make provision in his design of the steel frame to resist these horizontal forces and reliance must be placed upon some form of metal bracing to carry wind stresses. This may be obtained by means of rigid connections and special bracing members. In many high buildings, for example, the Woolworth Building, the entire wind stresses are carried by the wind bracing, no reliance being placed upon walls or partitions, except parallel to the long side of the building. The tower was designed independently as if standing alone.

The steel frame is generally run up ahead of the walls and partitions. In several instances the frame work has been wrecked during erection; in other cases it was found to sway under wind pressure, making it necessary to put in temporary bracing to stiffen the framework. The steel frame of a building should be treated as an independent structure the same as the towers of a viaduct, and should be able to resist the wind forces on all surfaces exposed during erection. This should be accomplished by substantial bracing or by designing the column and girder connections so that they may be able to resist the bending stresses produced by wind pressure. Mr. C. C. Schneider specifies that the steel frame work shall be designed for a wind pressure of 30 lbs. per sq. ft. on all exposed surfaces composing it, and the frame work shall be considered as an independent structure without walls, partitions or floors. In proportioning the members of the structure for these temporary wind strains it is permissible to use higher unit stresses than for permanent work, *e. g.*, 20,000 lbs. per sq. in.

The Municipal Building, New York, is 25 stories high and has large lateral dimensions. No special bracing was used but reliance for resistance to wind pressure was placed on its inertia, the stiffness of its girder connections, and the strength of its hollow tile floors. The curtain walls were begun several stories up from the curb. After the curtain walls had been placed in for a few stories above this level the framework began to sway under wind pressure and it was necessary to fill in the lower stories before erection could be continued. This shows that the steel framework must be proportioned for wind stresses during erection.

Observers will have noticed cross sections of buildings of similar dimensions and almost identical in character and general planning—where one structure is made to depend upon cross-partitions, end walls and the ordinary girder connections, with columns for trans-

Plate-I

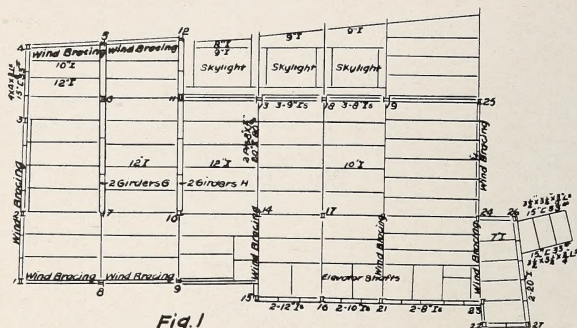
TYPICAL FRAMING PLANS SHOWING
PLAN LAYOUT OF WIND BRACING

Fig. 1

Third Floor Beam of the Trust Company of America Building showing plan layout of wind bracing.

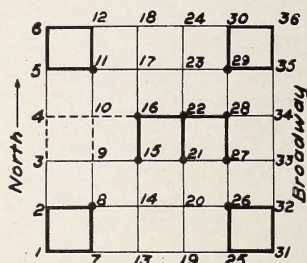


Fig. 2

Plan of Wind Bracing in Singer Bldg.

Heavy full lines wind bracing to 33rd story on corners, to 36th story on interior. Heavy dotted lines indicate wind bracing to 14th story (main bldg). Fine lines indicate no diagonal wind bracing. Circles on column intersections indicate anchor bolts.

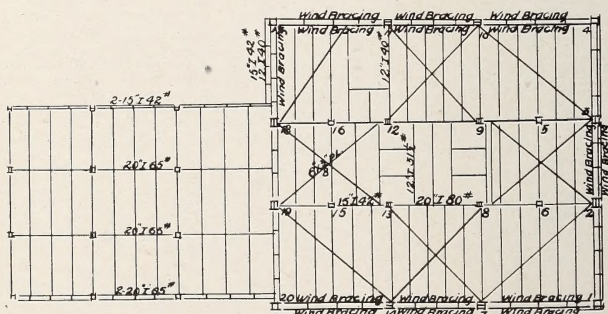


Fig. 3 Beam Plan of Metropolitan Tower

ferring horizontal stresses to the foundations, while the other has a distinct system of structural bracing capable of caring for similar forces. It would seem that either one owner was put to unnecessary expense or that the other has not proper insurance on his structure. The question naturally presents itself, whether modern steel buildings are being constructed with the same factor of safety against the various forces to which they are subjected, whether vertical, horizontal, or otherwise. From an engineering standpoint it seems reasonable to provide for each of the destructive forces to which it is subjected to a degree at least proportionate to their probabilities.

DISPOSITION OF WIND BRACING

The bracing, no matter which type is used, should be vertical and reaching down to some solid connection at the ground. It should be arranged in some symmetrical relation to the outlines of the building. For example if the building is narrow and is braced crosswise with one system of bracing it should be braced midway between ends of the building. If two systems are used they should be equidistant from the ends. The symmetrical arrangement is necessary to secure an equal service of the systems and prevent a tendency to twist.

It is believed to be economical to brace each transverse bent. The girders must be designed to act as wind bracing and the columns proportioned for both axial and bending stresses from wind. If the ordinary girder connections are used in a transverse bent, it does not seem likely that the columns on these sections can receive any appreciable wind stress, either axial or bending. Each braced bent must be designed for the full force of the wind contributory to the area under its influence. The proper selection of sections for the economical disposition of wind bracing must remain a matter of judgment.

In PLATE I. some typical framing plans, showing plan lay out of wind bracing, are shown. *Fig. 1* shows plans of bracing in the Trust Company of America Building. The bracing is symmetrically arranged, and as knee bracing is used it is mainly placed in the exterior walls so as not to interfere with the floor space. *Fig. 2* shows plan of bracing in the Singer Building. The wind stresses are resisted by 25 panels of vertical diagonal bracing between pairs of columns. A continuous system is used in the interior, while each corner has four braced panels. *Fig. 3* shows the disposition of the bracing in the Metropolitan tower. A continuous system of detached and solid web knees were used and placed entirely in the exterior walls. Horizontal X bracing was placed in the floors to distribute the wind loads to the interior columns.

WIND BRACING BY RIGID CONNECTIONS WITHOUT DIAGONALS, KNEES, GUSSETS OR PORTALS

In buildings of relatively large dimensions, in which the ratio of height to width is small, sufficient lateral stiffness to wind pressure

may be secured through rigid column and girder connections without introducing special wind bracing in the steel frame.

The following types are effective in producing greater rigidity at the joints and connections.

Type 1.—Continuous column splices.—Considerable stiffness may be secured by using continuous column splices, where the columns are made in two story lengths and staggered as to splices, i.e., only every alternate column is spliced at a floor. In present practice, however, all columns are usually spliced at the same floor, thus facilitating erection. When column splices are properly made they may be considered stronger than the main section of the column. The splice should be of sufficient strength to take the shear and flange tension due to bending.

Type 2.—The use of beam connections in framing beams and girders to columns.—The ordinary method of framing beams and girders is by means of shelf angles and column brackets. This does not stiffen the frame to any extent. A much more effective method is secured by using the ordinary beam connections in framing the beams and girders to the columns. This type of bracing was used in the upper eight stories of the Dominion Bank Building, Toronto. The beam connections stiffen the frame during erection as well as after the building has been completed. A stiffer frame will result if the beams run transverse to and the girders parallel to the longest dimension of the building. A beam should come at each column in order to give lateral stiffness to the frame.

Type 3.—Greater rigidity may be secured by using beam connections and also rivetting the flanges of girders and beams to the columns by means of corner angles. The usual corner angles are 8 x 6 in. or 8 x 8 in. Buildings are not ordinarily figured for wind longitudinally; this type seems to be amply suited for bracing the building in that direction. This method has been employed in the United Fire Companies building, New York, and the Continental and Commercial National Bank Bldg., Chicago.

It is known that beam connections are capable of transferring only small bending moments and that the columns can take only a small wind stress, either axial or bending. Mr. Forchhammer states that beam connections are not rigid but flexible. As to whether rigid or flexible connections are preferable in steel frameworks is a matter of some importance. The rigidity obtained by rigid connections is followed by the uncertainty of statically indeterminate structures. The stresses due to uneven settlement of columns for instance, may run very high in rigidly connected frameworks. Mr. Forchhammer favors the use of rigid connections at the side columns and flexible connections at the inside columns. However, to concentrate all the wind resistance in the outside columns does not seem advisable as the effect of uneven settlement might be more serious in this case, for in the outside, columns in which the whole stiffness of the frame lies might be seriously over strained. It would seem that a stiff connected frame is the only proper and practical solution of the problem.

Type 4.—By deep web connection angles.—Sometimes it is advantageous to obviate special bracing as diagonals or knees, which tend to obstruct the clear floor space and which require difficult connections. A good substitute may be had if floor girders and beams are web-connected to the columns so as to give very deep and rigid joints that can be relied upon to develop sufficient stiffness to resist distortion from wind pressure. In the City Investment Building, New York, deep heavy connection angles were shop-riveted to the columns and field rivetted to the webs of the I beams.

Type 4.—Continuous wall girders.—In this type the wall girders are made continuous across the face of the column for the whole length of the building. The webs of the girders are carried past the flanges of the columns, the inner flanges being cut flush with the web. The connection between the wall and the girder is made by a deep gusset plate rivetted across the face of the column and serving both as a splice plate and a knee brace against wind stresses. The gusset plates are usually connected by heavy connection angles to the outside of the column flange, the gusset plate being parallel with the column web.

This style of wind bracing has the advantage of avoiding all tension on the head of the rivets and transmits the stress entirely through direct rivet shear. It is, therefore, theoretically advantageous and proves easy and cheap in construction and erection. In a 35 story office building in Seattle, rigidity against wind stresses is given to the building by a continuous belt of 30 in. wall girders running around the entire building. The gusset plate is field-rivetted to the girder webs and field connected to the flanges of the columns by heavy connection angles shop-rivetted on the outside of both the column flanges.

Type 6.—Horizontal X bracing or trussing in the floors is sometimes used to distribute the stresses among the interior columns, particularly if the vertical wind bracing is placed entirely in the exterior walls. The horizontal bracing used on the 4th and the 6th floors of the Commercial Bank Building, Chicago, consists of single and double angles laid above the floor beams and rivetted to connection plates at the columns. In most cases the gussets or connection plates cannot extend through to the columns, and are rivetted, therefore, to the connection plates on the 15 in. channels forming the lines of spandrel bracing at these points.

VERTICAL X BRACING

The simplest form of bracing is by a system of vertical X bracing in the panels between the columns and floor girders to transmit the pressures developed by the wind pressure to the foundations. The rectangular shape of a building can be effectively and economically preserved against distortion from wind pressure by means of diagonals. For this type of bracing the stresses are statically determinate and the ease with which the stresses may be computed and the design facilitated makes this type very desirable.

Advantages and Disadvantages.—Diagonal braces of structural

shapes, either angles or channels, make the stiffest bracing and should be used in a few bents of a tall building. Metal rods with pin connections have been used but modern practise shows a preference for structural shapes with rivetted connections. If metal rods are used they should be tight in every connection for if there is any play or movement possible between members it cannot be very efficient.

Architectural requirements limit the use of diagonal bracing, as it interferes materially with the window and door spaces, corridors and other features. A complete system of diagonal braces cannot be placed in the outside walls on account of the numerous windows. It can often be arranged in the interior walls and partitions with no inconvenience to the design of the building.

Diagonal bracing imposes the condition that a comparatively thick partition be placed in the plane of the bracing for cover and protection of the steel. In the ordinary office building this condition divides the floor surface into box-like suites, usually of the same shape and size. No large opening, no freedom in the selection of a position for an opening or corridor—in short, no effective architectural medium—can be used for joining adjacent suites through such a partition. Usually the doors must be small and also in the centre of a panel or in one side, according to the type of diagonal bracing used. Corridors must either extend along the wall, thus cutting off a great deal of window light or down the centre of a panel, creating a row of offices on each side of the hall. These limitations will often result in very shallow offices and sometimes in no direct communication between suites.

ANALYSIS OF STRESSES

Diagonal bracing is essentially a two column or single panel type of bracing, the usual practice being to brace odd panels rather than a continuous system. Such a braced frame is usually regarded as a cantilever truss fixed at the ground by its own weight. The diagonal members are similar to the web members of a truss and the columns act as the chords of the wind truss. The columns may be either in tension or compression, according to the direction of the wind and in the latter case it is added to the static load.

The wind pressure is assumed to act horizontally and applied at the panel points.

The exposed area tributary to a panel point is equal to the sum of one-half the story height at the point plus one-half the height of the story below, multiplied by the width of the area affecting the bracing in the panel under consideration.

From a study of a stress diagram it will be seen:

(1) That the compression (or shear as it is commonly called) in any floor girder is equal to the load at that point plus all of the panel loads above.

(2) The stress in any diagonal is equal to the shear in the floor girder above multiplied by the secant of the angle which the diagonal makes with the horizontal.

(3) The compression in the upper story leeward column is

equal to the vertical component of the stress in the diagonal. The compression in the leeward column of any story is equal to the vertical component of the stress in the diagonal in that story plus the compression in the column of the story above. The vertical component of the stress in the diagonal is equal to the shear in the floor girder at the top of the story multiplied by $\tan$ of the angle between the diagonal and the horizontal. If we denote this vertical component by the term increment, then the compression in the leeward column equals the increment for that story plus the compression in the story above.

(4) It can be seen that the tension in the windward column in any story is equal to the compression in the leeward column in the story above.

(5) The uplift at the base of the windward column is the quotient of the moment of the resultant wind and the distance centre to centre of the columns. If this uplift exceeds the weight of the frame and its loads, anchorage of the column to the foundation will be necessary.

PLATE II shows typical details of diagonal bracing in the Dominion Bank Building, Toronto.

The column stresses from wind on this type of bracing are direct and statically determinate. In practice the diagonal members frequently cannot be designed so as to have the gravity axis of the columns, girders and diagonal members meet in a common point and some bending stresses result in the columns and girders. The floor girders are usually shallow and though the connections are rivetted they cannot be said to be rigidly connected, and the bending moments in the columns and girders due to whatever rigidity these connections may have are neglected. With diagonals, the stresses in theory are all direct, and there is no bending, with rivetted connections. However, as is usually the case in buildings, there will be secondary stresses due to the elongating and shortening of the members that take direct stresses.

The tensile stress in the windward column must not exceed the dead load plus a small live load; otherwise anchorage must be provided to resist the upward reaction, and the column spliced for tension. When this occurs the limit of efficiency of the bracing is reached, as it is impracticable to anchor or splice the columns. The dimensions and weight of a building are usually such that, considered as a whole, its moment is sufficient to give it stability against overturning, but in rare cases the margin has been so small that additional security has been provided by anchoring some of the columns to the foundations, notably in the case of The Singer Tower in New York.

WIND BRACING WITHOUT DIAGONALS

In a steel frame building with rectangular panels, i. e., without diagonals, its stability is dependent on the bending resistance of its various members. A correct determination of the stresses due to vertical floor loads as well as to horizontal wind forces, requires a consideration of the deformations of all the members of the frame.

Hence, the stresses are statically indeterminate and the stresses in each member are functions of all those in all the other members.

The rigid solution is too long and cumbersome for practical use in actual designing and approximate methods must be resorted to. For simplicity it is customary to figure the wind stresses independent of the direct stresses from vertical loads. This practise has led to the development of several approximate methods each with specific assumptions as to distribution of direct stresses and column shears. Some of the assumptions made for determining wind stresses will not hold under vertical loads. No assumptions that can be made will give correct stresses when the columns and girders are proportioned for direct loads and bear little or no relation to the stresses induced by the wind.

It is common to consider a transverse bent of a steel frame building as a cantilever beam, uniformly loaded, the columns acting as flanges and taking a part of the vertical reaction proportional to their distance from the neutral axis.

LOCATION OF THE NEUTRAL AXIS

Before the direct stresses in the columns can be determined it is necessary to locate the neutral axis of the bent.

The neutral axis is determined by the column spacing and their sectional areas, and it is found independent of vertical loads. It can be readily computed, if we consider vertical loads, when we know the magnitude and point of application of column loads on the footings. This neutral axis will not be coincident with the centre line of the building, unless the loads are symmetrically applied to the footings. In a steel frame building this is but seldom the case. A heavy spandrel wall on one side and a curtain wall on the other, unequal loading due to elevator framing, stairways, vaults, etc., are some of the things which make it necessary to determine the neutral axis for each bent by independent calculations. Since the sectional areas of the columns may be taken as proportional to their loads it will be sufficiently accurate for all practical purposes to locate the neutral axis independent of column loads.

DIRECT STRESSES IN COLUMNS

When the neutral axis has been located then the direct stresses in the columns can be determined. As in a beam under flexure the fibre stresses vary directly as their distance from the neutral axis, so in a transverse bent the direct stresses in the columns will be proportional to their distances from the neutral axis. It is assumed that all columns on the windward side of the neutral axis take a direct stress of tension and those on the leeward side a direct stress of compression.

Column Shears.—A common assumption is that the horizontal shear on any plane is equally distributed among the columns cut by that plane. This is true if each column has the same moment of inertia, but if the moments of inertia are different the horizontal shears taken from the columns will vary as their moments of inertia.

WIND STRESSES IN RECTANGULAR BUILDING FRAMES

The common methods for computing the wind stresses in the main members of a steel frame building with rectangular panels will be given and the method modified later where knees, plate girder, or portal bracing are used.

Cantilever Method.—This method was developed by A. C. Wilson and explained in detail in the *Eng. Record*, Sept. 5, 1908. It is slightly modified by R. Fleming in *Eng. News*, Mar. 13, 1913. The following statements are taken from Mr. Wilson's article entitled, "Wind Bracing with Knee Braces or Gusset Plates."

"If a beam of rectangular section be loaded as a cantilever with concentrated loads, it is possible by the theory of flexure to find the internal stresses at any point. If, however, rectangles be cut out of the beam between the loads, there will be a different condition of stress. What was the horizontal shear of the beam will now be a shear at the point of the contra-flexure of the floor girders, causing bending, and, as in the beam, the nearer the neutral axis the greater the shear. The vertical shear in the beam would be taken up by the columns as a shear at the points of contra-flexure and the amount of shear taken by each column would, as in the beam, increase towards the neutral axis. The direct stress of tension or compression in the beam would act on the columns as a direct load of either tension or compression, and, as in the beam, would decrease towards the neutral axis.

"Each intersection of column with floor girders would be held in equilibrium by forces acting at the points of contra-flexure; and to find all the forces acting around a joint at any floor the bending moments of the building at the points of contra-flexure of the columns above and below the floor in question are found as will be explained later.

"It is assumed that if a beam of constant, symmetrical cross-section and homogeneous material is fixed at both ends, and that if forces tend to move those ends from a position in the same straight line to a position to one side with the ends still parallel, reverse bending will occur with the point of contra-flexure in the centre of the unsupported span. And since this condition exists in all columns and floor girders it will be necessary to find the shears at the points of contra-flexure as well as the direct stresses in all the members."

In order to find all the internal stresses it is necessary to find the total horizontal shear and overturning moment of the wind at the line of contra-flexure of each story columns.

After finding the position of the neutral axis and the direct stresses in the columns, the method of finding the column and girder shears and moments will be the same for the general as for the special case in which the sectional areas and column spacing are equal.

PORTAL METHOD "A" (COLUMN SHEARS EQUAL)

Assumptions.—(1) The bent is assumed to be a series of independent portals.

(2) The shears taken by each column are proportional to their moments of inertia.

For equal column spacing and assumed equal moments of inertia, the interior columns have their vertical components neutralized by the equal stress of opposite direction caused by the contra-flexure of the columns, and the outside columns take all of the vertical reaction of the bent. If the spacing between columns is unequal, the direct stresses from adjacent panels are unequal. The resultant is a direct stress between the two portals considered.

The direct stresses in the interior columns are zero and outside columns only have direct stresses. The direct stress in the outside columns at any story is equal to the overturning moment about its point of contra-flexure divided by the width of the bent.

PORTAL METHOD "B"

It is assumed that the interior columns take twice as much horizontal shear as the outside columns. This assumption gives a much better distribution of metal and seems to be a more rational assumption than that of method "A."

If the building has self supporting walls and the outer columns carry floor loads only, assuming the moments of inertia of the outside columns to be one-half that of the interior columns, the shear taken by the outside column will be half of that of the interior columns and the problem is similar to the preceding one. It will be observed the moments calculated by this method compare favorably with those calculated by the cantilever method.

A justification of the assumption made in this method is here-with given. In a rolled beam or plate girder it is common to assume the shear as uniformly distributed over the entire cross section. The intensity of the shear at any point is equal to the total shear divided by the area of the web plate. The portion of the total shear taken by any section of the plate would then be equal to its depth divided by the depth of the girder.

KNEE BRACING

Types.—1. Detached Knee Braces.

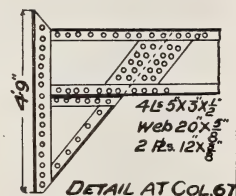
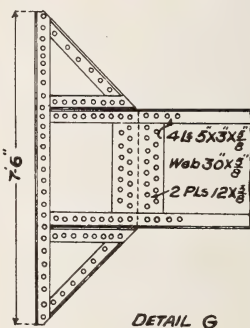
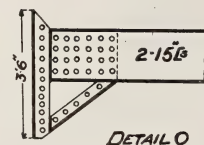
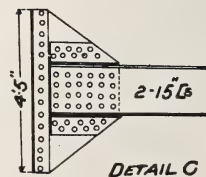
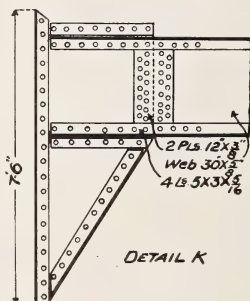
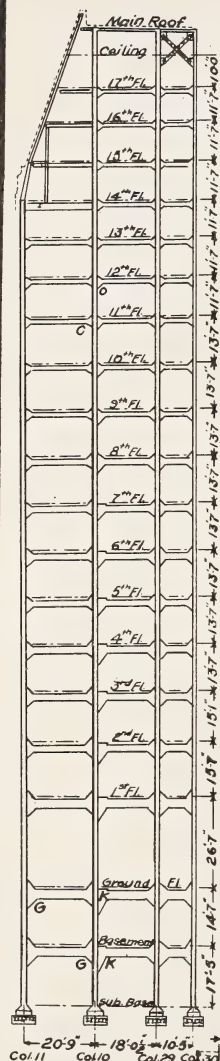
2. Solid web knee braces, brackets, or gusset plates as they are commonly called.

Detached Knees.—Detached knees are usually built of structural shapes, either angles or channels. Where the stresses are small the brace may consist of a single angle or two angles back to back as in the Dominion Bank Building. For moderate stresses a single channel or two channels back to back may be sufficient. In the lower stories of tall buildings where the stresses are large, very deep and heavy knees must be used if there is to be any considerable reduction in bending stresses.

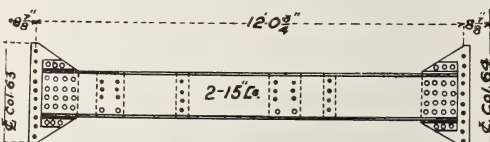
Solid Web Knees.—A very efficient wind brace member is a deep plate girder at each floor level with a special end connection either by means of large gusset plates or an extension of the column web.

PLATE-III

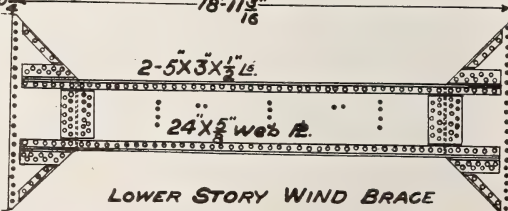
WIND BRACING IN THE PLAZA HOTEL, N.Y.



WIND BRACE CONNECTIONS



UPPER STORY WIND BRACE



LOWER STORY WIND BRACE

TYPICAL WIND-BRACING PANEL

The gusset plate is well illustrated in the Plaza Hotel, New York on Plate III. The girder web is cut short at each end and a special end connection made with a large gusset plate shop-spliced with double splice plates to the girder web. The gusset plate extends above and below the top and bottom flange of the girder forming double knee braces integral with the girder. The gusset plate may be field-riveted, either directly or by means of connection angles, to the column flange. The gusset plate is usually stiffened with 3 x 3 in. angles, and the upper and lower flanges of the girder are shop-riveted with connection angles to it.

In the Woolworth Building, New York (see Plate IV, Figs. 4 and 5) the gusset plate is shop-riveted to the columns and field-riveted with double splice plates to the girder web. The bottom flanges of the girders are field-riveted both to gusset plates and to horizontal shelf angles shop-riveted to the gusset plate. This arrangement provides for convenient support and assembling during erection and makes all the field-driven rivets accessible.

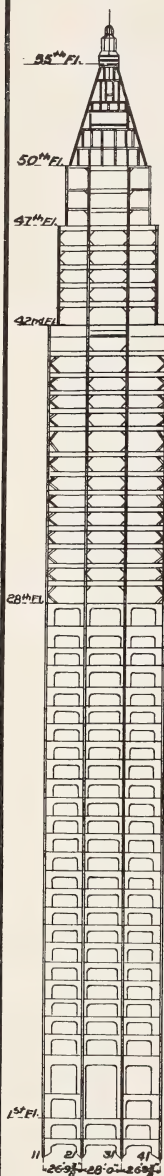
In the Union Central Life Insurance Building, Cincinnati, the column web is extended to make the connection. The girder web is cut short to clear the projecting section of the column web which is field-riveted to it through double splice plates. The upper flange of the girder is connected to the column web by connection angles, shop-riveted to the flange and field-riveted to the column web. The lower flange engages the column web through connection angles, field-riveted to the flange-angles and shop-riveted to the column web. These horizontal angles serve as brackets for the girders during erection.

When the solid web type of knee brace with deep plate girders is used the girders must be rigidly connected to the columns. This can only be accomplished by properly connecting the flange angles as well as the girder web to the column. The large bending moments at the connection require an extension of the column web to make the connection properly. This type of connection is expensive and difficult to make in the field. The best substitute is the gusset plate connection, but this is not as satisfactory as an extension of the column web.

Discussion on Knee Bracing—Knee bracing, whether detached or solid web knees, while an effective and efficient method, is not often a practical method of bracing a building, except in the end or wall sections of a building because of the interference of the knee with the floor space.

Knee braces do not appreciably diminish the bending in the columns unless placed both at top and bottom of girder; and for economy double knees should be used. In general no advantage is gained by making top and bottom knees of different depths. If knees are used only at top of story they should be deep and heavy. The deeper the bracing the greater will be the reduction in bending stresses. If the double knees be deep enough to intersect the column at mid-story height, the bending in the columns will be reduced to

Plate-IV



Wind Bracing
in Tower
Fig. 1

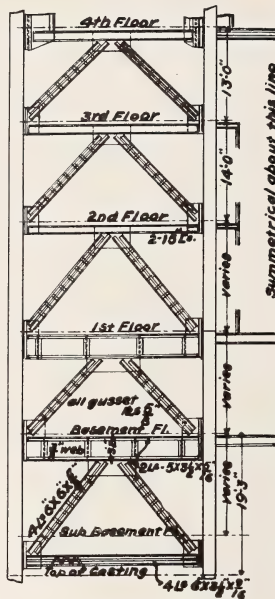


Fig. 2

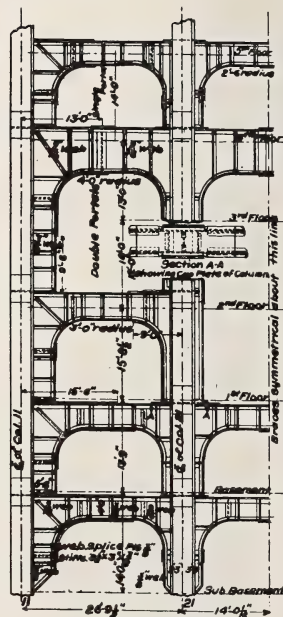


Fig. 3

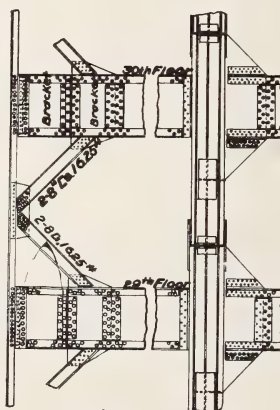


Fig. 4

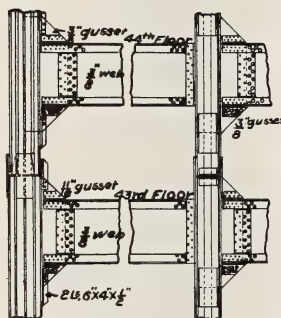


Fig. 5

Wind Bracing in Woolworth Building New York

zero. The same is true of the girders if the knees intersect the girder at mid-span.

When the solid web knee brace is used, the bending stresses will be carried mainly by the stiffening angles. Although the solid web knees are not usually deep, they are usually stiffened by 3 x 3 in. angles; it is recommended that if the knees are made deep, the angle stiffeners should be made about half as heavy as when they compose the entire brace. The extra cost of the solid web knee brace is warranted, as the extra metal is not lost; it gives a very rigid joint, takes care of the large bending moments at the connections, and besides performs the function of a knee brace.

Portal Arch.—Where exceptional stresses require heavy bracing the portal arch has been used, notably in the Woolworth Building.

The Portal Arch consists of a solid web plate out of which the arched opening is deducted, stiffened by angles bent to the curve of the opening. The bracing is field-rivetted by means of connection angles to the flanges of columns and girders. In the Woolworth Building (see Plate IV, Fig. 3) the portals were proportioned for the combined stresses due to live and dead and wind load. The portal webs were field-rivetted to the columns through the projecting cover plates of the latter.

WORKING STRESSES FOR WIND PRESSURE AND SPECIFICATIONS

The impracticability of using diagonal braces in the majority of buildings makes it necessary to have recourse to solid web or detached knee braces, and deep plate girders to provide the requisite lateral stiffness in at least some directions. When the diagonal is removed its stress is taken up by bending in the columns and floor girders. The columns and floor girders must be designed to carry the resulting and combined stresses.

The method of combining bending with direct stress in such members as columns and floor girders is not simple, involving as it does functions of the correct section (section modulus) and is necessarily a matter of trial. It is difficult to assign judicious working stresses in such a combination especially as there are practically no experimental data to guide the judgment. It is practically certain that the greatest permissible stress where bending is involved may be greater in a member than where a stress in a member is all direct, for in the former case the greatest stress exists along a line and not uniformly over the entire cross section. It is an open question that an excess of 10 to 20 per cent. may be permitted.

Most building by-laws have the common specification of 30 lbs. per sq. ft. for wind pressure, on the actually exposed surface and permit an increase of 20 to 50 per cent. in working stresses for combined stresses due to wind, live and dead loads; but the section shall not be less than if the wind forces be neglected. Chicago and San Francisco specify 20 lbs. per sq. ft. with an increase of 50 per cent. in working stresses. C. C. Schneider in his "General Specifications for Structural Work of Buildings," calls for a wind load of 20 lbs. per sq. ft., and allows an increase of 25 per cent. for bracing and combined stresses due to wind and loading. Ketchum's specifications for "Steel Frame Buildings" permit an increase of 50 per cent. in working stresses for combined stresses.

GEAR MATERIALS

C. B. HAMILTON, B.A.Sc. '06.

Some of the data which has been prepared for publication in the hand book of the Hamilton Gear and Machine Co., is of a nature to be of general interest to engineers. An intelligent choice of proper materials may, and often does, make the difference between success and failure. This applies with especial force to elements of machines which must resist two or more kinds of hard usage. Gears, for example, must resist static loads, shocks and also surface wear.

The whole tendency of modern practice is towards the use of high grade materials and better workmanship. In gears this means a smaller size for the same power with a consequent lowering of pitch line velocity. From this directly follows a reduction in noise and wear. Often it is possible thus to increase the speed of revolution of the whole mechanism and save doubly on the size of the various parts.

Cast Iron is the commonest material for gears. Its weak point is lack of toughness which makes it unsuitable for pinions. Steel pinions are frequently used to mate with cast iron gears. The teeth of the pinion are subjected to much greater stress than those of the gear. Semi-Steel is cast iron with the addition of a percentage of steel. It is stronger than cast iron.

Steel Castings make the strongest gears for large work but care must be taken to secure sound blanks as this material is subject to blow holes and shrinkage strains. Careful foundry work and annealing of the castings will overcome these difficulties. Open Hearth Steel should be used for gear work.

Steel Forgings are superior to steel castings both as to quality of metal and certainty of freedom from unseen defects. They can be made in a wide range of carbon contents giving a choice of hardness. This material is the best at moderate price for pinions.

Machinery Steel is similar material to that in steel forgings but is commonly finished in rolling mills instead of under a steam hammer or hydraulic press. Bessemer steel is not usually considered good enough for gears. Open Hearth should be used unless the job warrants the expense of crucible steel.

Alloy steels containing various percentages of metal such as nickel, chrome, vanadium, etc., are in common use for automobile and other high grade gears. They are produced by both the open hearth and crucible processes and are usually drop forgings. This material is expensive but of high quality, and is always heat treated to develop its superior qualities. There are two classes according to method of heat treating. First, case hardening steels which only harden after being pack carbonized and remain soft at the core. Second, tempering steels which, when heated and quenched will harden clear through and should

have the temper drawn down afterwards to give toughness. The former are used for rear axle bevel gears and the latter for transmission and clash gears.

Bronze is a good and reliable material. Brass is of very little use except for the flanges of rawhide pinions. For this work a good red brass is suitable. For gears or pinions it is best to pay a few cents more per pound for No. 1 Government Bronze, 88 C, 10 T, 2 Z, or a phosphor bronze of similar proportions.

For worm wheels the 2 Z is sometimes replaced with 1% Zinc and 1% Lead. Cheap worm wheels are made of C. I. and driven by steel worms. High grade work calls for bronze wheels and case hardened steel worms. The best worms are hardened and ground.

For silent drives, rawhide, fibre or cotton is used for the smaller gear. These materials are held between flanges, riveted or bolted together. Rawhide or Cotton is about equal in strength to Cast Iron, while fibre is about half as strong.

Large gears, especially bevels, are sometimes made with wooden teeth for smooth running. The smaller gear would be cast iron with the teeth about 35% of the circular pitch in thickness. The larger gear would have maple cogs set in mortices in the cast iron body and fastened with keys on the back.

Gears running together of like material, especially if it be soft, wear rapidly, just as in the case of a journal and bearing. This does not apply to cast iron, which forms a glaze or skin in service.

On October 8th Mr. W. G. Chace, B.A.Sc. '01, Chief Engineer of the Greater Winnipeg Water District, addressed a meeting of the Manitoba Branch of the Canadian Society of Civil Engineers. He explained many of the features of the Shoal Lake water project, now under way and traced the efforts of the city of Winnipeg to obtain an adequate water supply, leading up to the adoption of the present undertaking.

Mr. C. H. Mitchell, C.E., '92, who is with the first contingent, has been promoted to the rank of Lieutenant-Colonel. He has also received an appointment to the position of General Staff Officer on the Headquarters' staff of the first contingent, which is one of the highest appointments given to a militia officer in Canada.

B. H. Hughes, '16, on account of illness, was unable to sail with the first contingent, but will accompany the second contingent when they leave for Europe.

We understand that Messrs. H. A. Heaton, '15, G. B. Taylor, '14, and L. C. M. Baldwin, B.A.Sc. '13, who were in England when war was declared, have enlisted with the forces there.

SWORD-ISMOND

On Monday, October 12th, Mr. A. D. Sword, B.A.Sc., '08-'09, of the engineering staff of C. W. Noble, Toronto, was married to Miss Lillian Fredericka Ismond, Toronto. Mr. and Mrs. Sword will reside at Clarkson, Ont.

WILSON-BARBER

On Thursday, October 15th, Mr. H. P. Wilson, B.A.Sc., '14, superintendent of construction for the Elias Rogers Company, was married to Miss Eleanor Mae Barber, youngest daughter of Mr. and Mrs. Timothy Barber, Toronto.

Mr. T. Kennard Thomson, D. Sc., '86, has been appointed by Governor Glynn, of New York state, a member of the Atlantic Deeper Waterways Commission. This commission has under investigation a system of canals to extend from Florida to Maine, with a total length of between seven and eight hundred miles and at a probable cost of \$35,000,000.

J. B. Challies, B.A.Sc., '04, superintendent Water Power Branch, Department of Interior, Ottawa, recently was elected an associate member of the American Society of Civil Engineers.

J. M. Duncan, B.A.Sc., '10, has been commissioned engineer-lieutenant, R.N. on H.M.S. Victory for the duration of the war.

E. A. Twidale, B.A.Sc., '14, who is color-sergeant, F Company, 44th Regiment, is at Allanburg, Ont., with the guard on the Welland Canal.

C. C. Rous, B.A. Sc., '13, is with the corps of engineers, established by the Canadian General Electric Co. for service at Quebec, Can.

R. A. Campbell, '09, has resigned his position as superintendent of the Tagona Water & Light Company, Sault Ste. Marie, Ont. We understand that he is succeeded by Mr. R. D. S. Beckstedt, B.A.Sc., '09, who was formerly sales agent for the same company.

M. C. Hendry, B.A.Sc., '05, chief engineer Manitoba Hydrographic Survey, Water Power Branch, Department of Interior, addressed the recent Irrigation Congress in Calgary on the power and storage investigations of the Department on the Bow River, west of Calgary.

S. A. Hustwitt, B.A.Sc. '14, is employed at Homer, Ont., on Section No. 2 of the Welland Ship Canal.

The following "School" men, in addition to those mentioned heretofore in "Applied Science," accompanied the first Canadian Overseas Expeditionary Force: N. R. Robertson, B.A.Sc. '06, M. B. Watson, B.A.Sc. '10, R. V. Macauley, B.A.Sc. '11, A. G. Gray, B.A.Sc. '13, C. B. Ferris, '13, W. W. Code, '15, D. H. Storms, '15, C. P. Cotton, '15, G. L. Magann, '15, Hal. Wallace, '15, Hugh D. Wallace, '16, R. W. Harris, '16, A. P. MacLean, '17, T. S. Glover, '17, F. L. Eardley-Wilmot, '17, P. C. De Gruchy, '17.

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EDITORIAL

THE WAR AND THE ENGINEER

War has an interest for the engineer which it cannot have for the layman or for men of other professions. Engineering is a profession on which the country relies, alike for its machines of war and of peace, from a battleship to a barbed wire entanglement on the one hand, and from a power station and manufactories to a plough-share on the other. It suffers perhaps more severely than any other profession from the decreased activity in commercial enterprises and the cessation of construction and progressive improvement, caused by the great international conflict. The engineer has reason to be, and is as eager as anyone to avert war, but now that the machines and

implements of his making have been brought into action in defensive and offensive capacities, the great struggle affords him a field for study which does not present itself to the other professions.

The four chief elements which make for success or failure in war are, (1) The resources of the country concerned; (2) The materials and implements of warfare used; (3) The officers and men engaged; (4) The statesmen who control the policy. To the engineer falls the duty of developing the natural resources of the country and the machines of warfare are the finished product of his efforts to render those resources of greatest service. He also contributes his share toward the latter two elements which play so important a part in determining the destiny of a nation.

The engineer was originally a soldier, the builder and destroyer of fortresses, the maker and user of cannons, and to him war has always presented a distinctive field for scientific study regarding the efficiency of his productions, and for a scientific test of the theories applied in the embodying of new materials in new offensive and defensive machines of war.

In the short interval that has elapsed since the South African war, he has given many new things to the army and navy, and while entertaining a horror for the destructive operations of such a gigantic conflict, he seizes the opportunity to learn where weaknesses are to be found, to compare the strength and value of different machines, and to determine where improvements can be made. The South African war has taught us many things about field artillery and those lessons are embodied in our new 18 lb. field piece which we believe is superior to any others in the world. It will be determined during the war whether the smaller field gun, which throws a 15 pound shell, makes up in muzzle velocity for what it lacks in weight. The carriages for the field guns have also been greatly modified during the last few years. The old form of carriage that recoiled with the gun has given way to the modern field gun-carriage, which is an elaborate structure with recoil cylinders and running-out springs. The gun runs back with but little movement of the carriage and the rapidity of fire is greatly increased. Engineers are waiting anxiously to know how great a part this highly developed weapon will play in deciding the issue of battle. The value of the airship as a machine of war will now be decided once and for all. Mechanical transport is also undergoing its first real test in the actual theatre of war.

So far our fleet has not had an opportunity of determining what value should be placed on new designs or whether the theories upon which they are based are well founded. Before hostilities have ceased we will know whether the subtlety and secrecy of the attack of the submarine will render it the dangerous factor which we believe it to be. We will find out whether

the ingenuity of the engineer has made the torpedo as formidable as it seems, the deadliest and most fearful of naval arms. It will be of interest to know whether the turbines in use will be as reliable as the old reciprocating engines. In fact, there are hundreds of things which now have the keenest attention of the engineer, including the new uses of electricity, the development of wireless signalling, the new system of laying mines, and the relative strength of armour and guns.

There are other means by which the engineer can do much for the Empire. This present conflict is in reality a two-fold war. One is a war of arms and men on the battlefield; the other is a "Trade" war. The latter may yet prove to be the deciding factor in the final outcome of the struggle, and we are proud to know that while our armies have been doing such valiant service on the battlefield, our fleet has maintained supreme control of the highways of commerce. German and Austrian trade has been cut off from many countries and those markets are open to us now. We should act quickly and seize the opportunity before they have been supplied by other countries which are also on the alert. Every manufacturer desiring foreign trade should send out the best agents at his command to-day, for this is undoubtedly the psychological time.

The doctrine that we should use British-made goods has been preached rather than practised during the last few years. The policy of inter-purchasing between manufacturers in this country should be developed. One effect of the war will be to prompt engineers to specify and purchase more British-made materials than heretofore. The various municipalities of the country should assume the lead in this direction. The makers of such materials should do their share to reduce the cost as much as possible so that a continuance of the business will be assured. The policy of British-made goods for British firms can not be carried out in its entirety, but it can to a much greater extent than heretofore. So far as it is impossible to use British-made materials, we should show a marked preference toward friendly countries, including the kindred republic to the South of us.

When the war is over we will find ourselves in a new world; and Canada will emerge fresh and strong from an ordeal more severe than she has experienced before. New activity will assert itself in commerce and industry and we will have no more to do with people grown fat and listless from a long unbroken period of peace and plenty. The engineers who can maintain activity now will have a decided advantage when hostilities cease and the wheels of progressive industry and construction again assume a renewed vigor and unabated impulse. Manufacturers who keep their products before the public to-day while others are allowed to recede to the background and out of the mind of the consumer, will reap a rich return upon the resumption of industrial progress.

MILITARY ACTIVITIES AT THE SCHOOL

While men of all stations in life throughout the Dominion are eagerly enlisting to serve their flag and country, the men of our University are not idle. The same patriotic impulses and the same love of flag and freedom, actuates them. The spirit of unbounded patriotism prevades the atmosphere of the University, and the students and professors alike are eagerly preparing themselves for service if they are needed.

An application was made to the militia authorities for permission to form a Canadian Officers' Training Corps in connection with the University. At a mass meeting in Convocation Hall on Wednesday, October 21st, President Falconer read a communication from the Acting Adjutant-General at Ottawa authorizing the formation of such a corps. The object of the organization is to give the men a military training, sufficient to allow each man to qualify, if not as a commissioned officer, at least for service as a non-commissioned officer or private. No obligation to go to the front is entailed but if the opportunity comes, the men in training will be ready to respond. Over 1,500 men have already enlisted and new recruits are being added to the ranks daily.

The Engineering students are taking a very active part in the formation of the corps. On Wednesday, October 7th, a meeting of the Engineering Society was held in Convocation Hall to discuss what steps they should take toward preparing themselves for active service. Much enthusiasm was manifested and a great deal of discussion was indulged in. Colonel Lang explained the requirements in connection with the training corps if it should be authorized by the militia authorities. On Friday afternoon, October 9th, another meeting of the Engineering Society was called. It was arranged that the recruiting of the students in Applied Science and Engineering should be left in the hands of the Engineering Society. Of a total enrolment of 525 students, over 450 have enlisted for drill. The companies of the "School" are in charge of Captain A. D. LePan, B.A.Sc. '07, who has given a great deal of his time toward the organization of these companies. Drill has been in progress for over two weeks and marked progress has been made.

A course of lectures on military engineering will be given to the engineering students, including a study of the International Morse Code used by the signalling corps. This code besides being used in wireless telegraphy, is also employed in military signalling by lamp and heliograph, and forms the basis of signalling by flags.

All lectures and laboratory work throughout the University after 4 p.m. has been cancelled, to afford the staff and students an opportunity to take the drill. A great deal of assistance is given by officers from the Barracks, who assist daily in training the recruits.

OBITUARY

A. R. CAMPBELL, '02.

The sad news was received of the death of one of our graduates, Mr. A. R. Campbell, '02, on August 20th, when he was accidentally drowned in the Rideau Canal near Ottawa. Mr. Campbell was Senior Major in the 105th Fusiliers, Saskatoon, Sask., and was in Ottawa for the purpose of offering his services to the Government when the fatal accident befell him.

Deceased was widely and well known throughout Ontario, as well as Saskatchewan. After graduating in 1902 Mr. Campbell was located in Collingwood and Sault Ste Marie, Ont. In 1903 he became resident engineer for Mr. Willis Chipman of Toronto, and was employed in that capacity until the end of March, 1907. During 1904 he was located at Lethbridge, Alta.; in 1905 he was at Strathroy, Ont., Moose Jaw and Saskatoon, Sask., and during the remainder of his association with Mr. Chipman, he was stationed at Saskatoon. He afterwards started a general Contracting and Engineering business in Saskatoon and secured several large contracts in that city.

A widow and two children at Saskatoon survive to mourn his loss. We extend our sincerest sympathy in their sad bereavement.

H. H. Brown, B.A.Sc., '14, is with the Lackawana Steele Co. at Buffalo, N.Y.

E. S. Noble, B.A.Sc., '11, is with the Northern Ontario Light and Power Co. at Cobalt, Ont.

A. G. Christie, '01, formerly Associate Professor of Steam Engineering at the University of Wisconsin, Madison, has resigned and received an appointment as Associate Professor of Mechanical Engineering, in the new Engineering Department of Johns Hopkins University, Baltimore, Md.

C. S. Clendenning is in the Engineering Department of the Pacific Great Eastern Railway, at Quesnel, B.C.

DIRECTORY OF THE ALUMNI

Robinson, L. H., '04, was resident engineer for the T. C. Ry, at Superior Junction, Ont., when last heard from.

Robinson, W. A., '08. His address is 259 Young St., Winnipeg, Man. We understand that he is right of way surveyor for the C.P. Railway.

Robinson, R. C., '08. His address is 173 Yale Ave., Winnipeg, Man. He is assistant engineer in the Canadian Northern Railway Bridge Department, in that city.

Robinson, W. E., '11. His home address is Beathton, Ont.

Roblin, H. L., '11, is assistant engineer in the Cranbrook office of the Water Right branch of the British Columbia Government.

Roddick, J. O., '06, is assistant engineer in the department of public works of Canada.

Rogers, J., '87, is town engineer at Mitchell, Ont.

Rogers, C. H., '06, is with the Peterboro Canoe Co., Peterboro, Ont.

Rogers, L. J., '08, is demonstrator in chemistry, at the University of To-

ronto, in the Faculty of Applied Science and Engineering.

Rolph, H., '94, is secretary to John S. Metcalf Co. Limited, Montreal, Que.

Rolfson, O., '06, is taking a post graduate course in astronomy at the University of Toronto.

Rose, K., '88, is manager of the Evans Rotary Engine Co. of Canada, Curry Building, Toronto.

Rose, R. R., '08. His home address is 65 Paisley St., Guelph, Ont. When last heard from he was in the Lands Department, Lake Superior Corporation, Sault Ste. Marie, Ont.

Rosebrugh, T. R., '89, is Professor of Electrical Engineering at the University of Toronto, Toronto, Ont.

Ross, J. A., '92, is designer in the engineering office of the L. S. & M. S. railway at Cleveland, Ohio.

Ross, J. E., '88, is on a surveying staff of the Department of Interior at Kamloops, B.C.

Ross, D., '08, is on the staff of the Toronto Power Co., 10 Adelaide St. E., Toronto.

Ross, R. A., '90, is consulting electrical and mechanical engineer at 80 St. Francois Xavier St., Montreal, Que.

Ross, K. G., '06, is with Lang & Keys, engineers and surveyors, Sault Ste. Marie, Ont.

Ross, R. B., '05, deceased.

Ross, R. C., '06. We understand he is with the Department of Interior, Ottawa, Can.

Ross, O. W., '10, has been engaged on the Welland Canal survey. His address on our file is 56 Chestnut Ave., Brantford.

Rothery, L. W., '11, is with the Westinghouse Machine Co., at Turtle Creek, Pa.

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Rutledge, L. T., '09, is demonstrator in drawing at the University of Toronto.

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S

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